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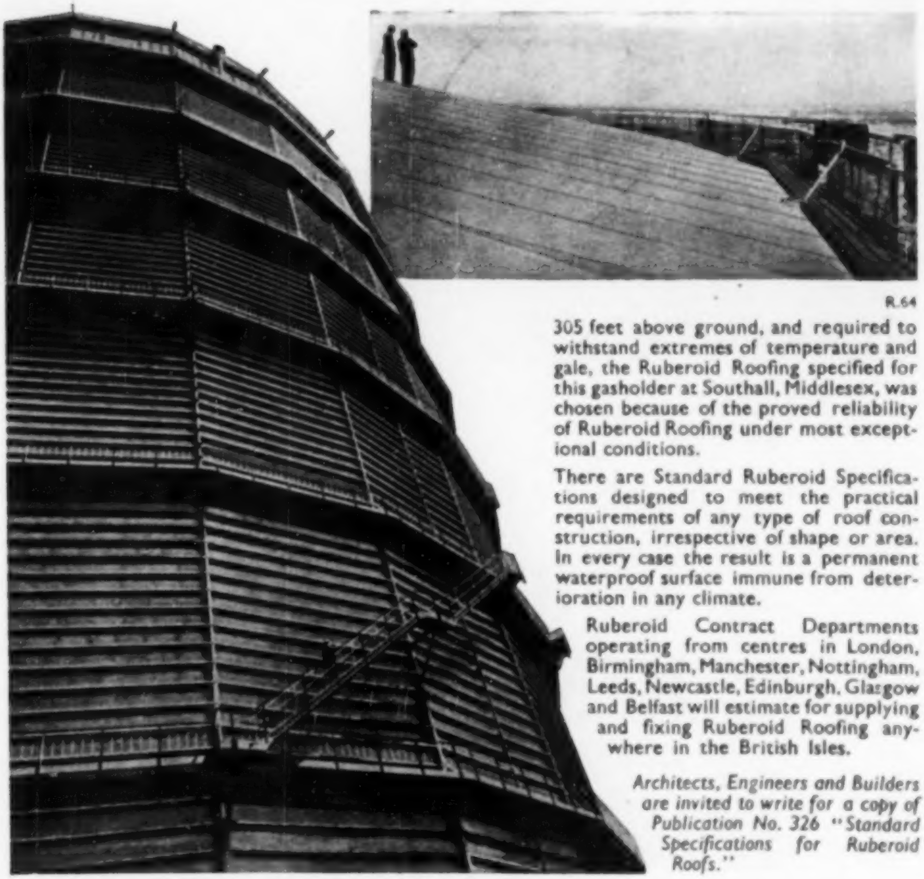

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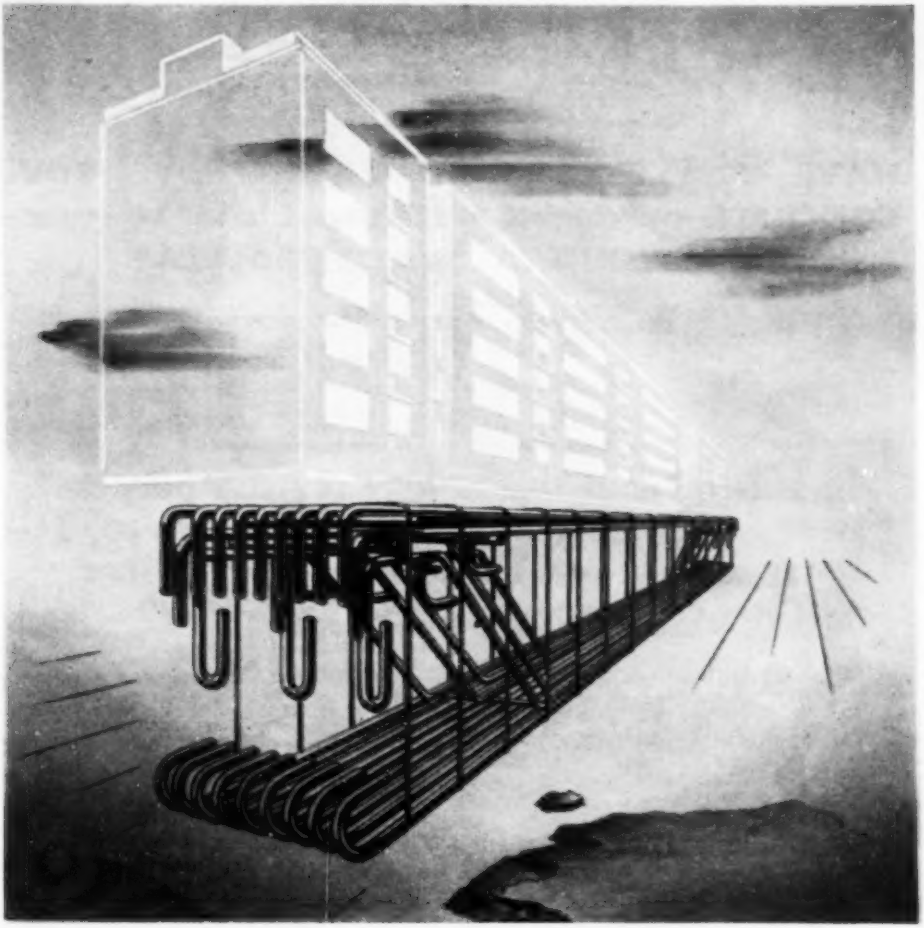

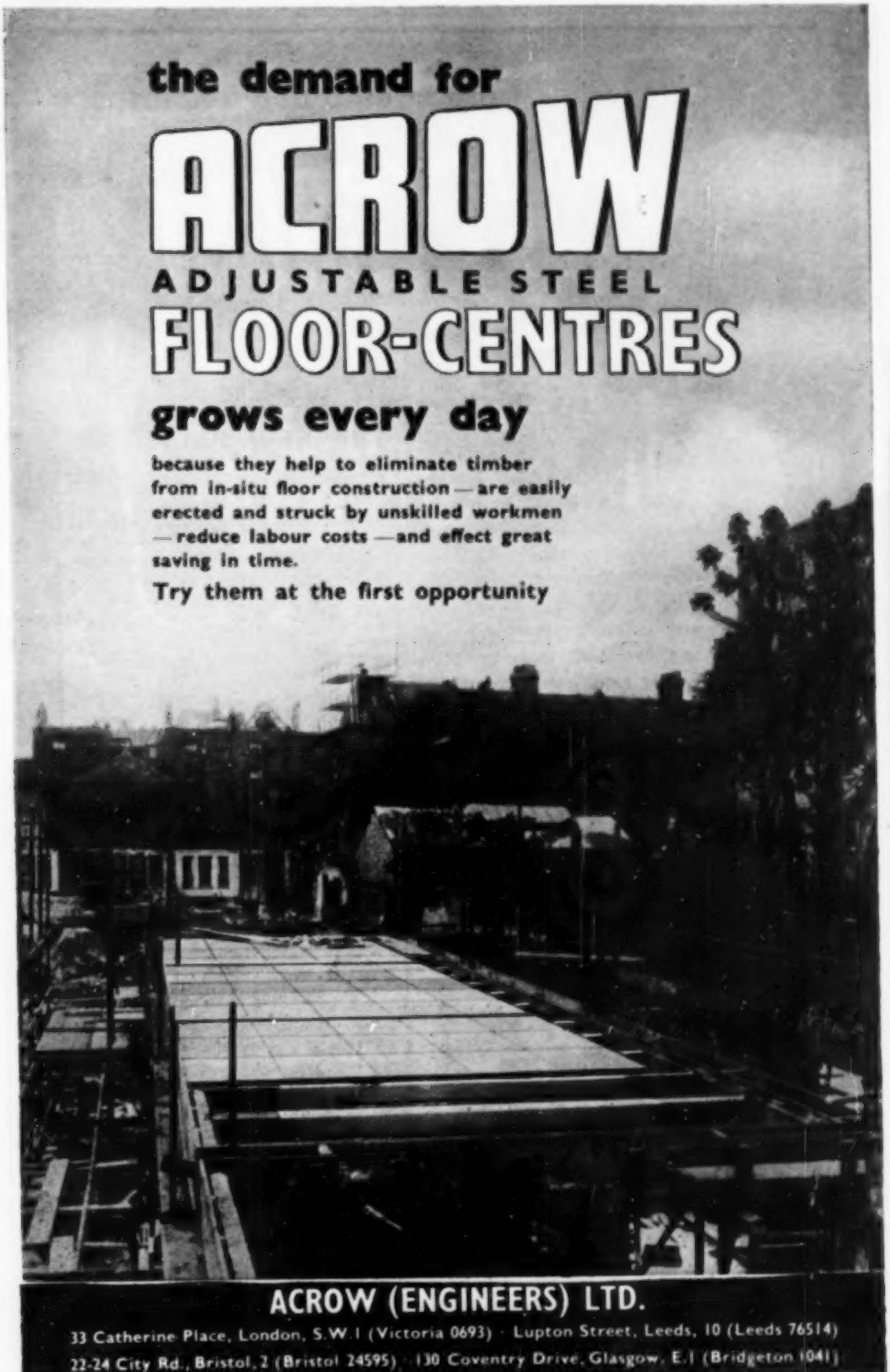
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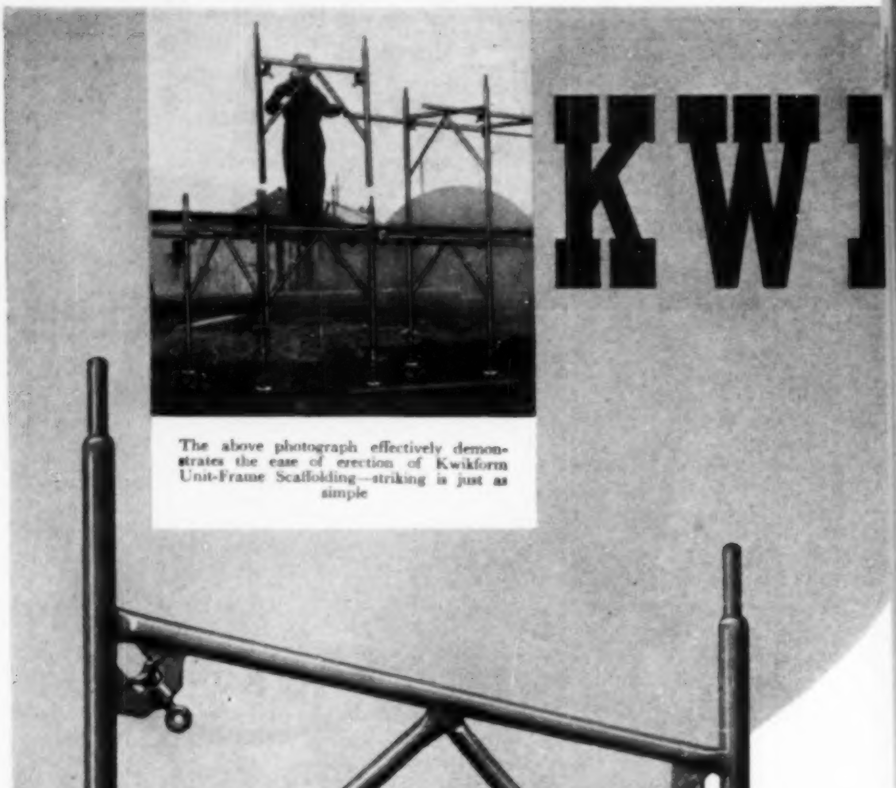

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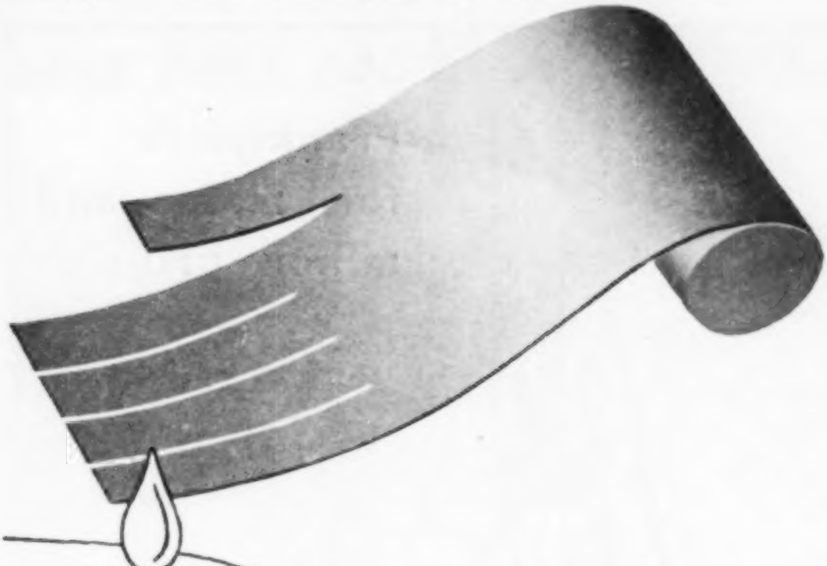

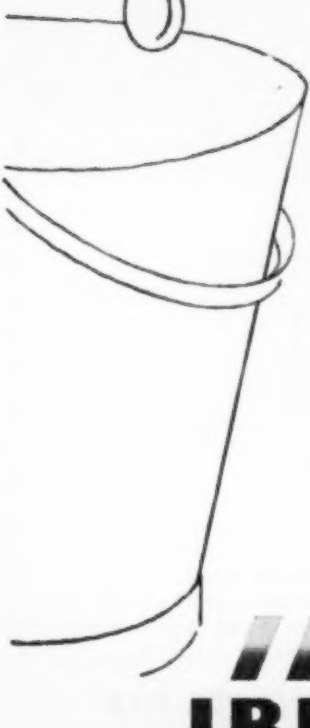

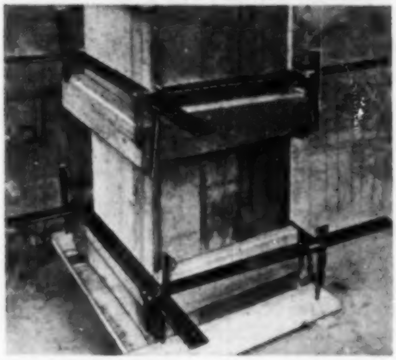

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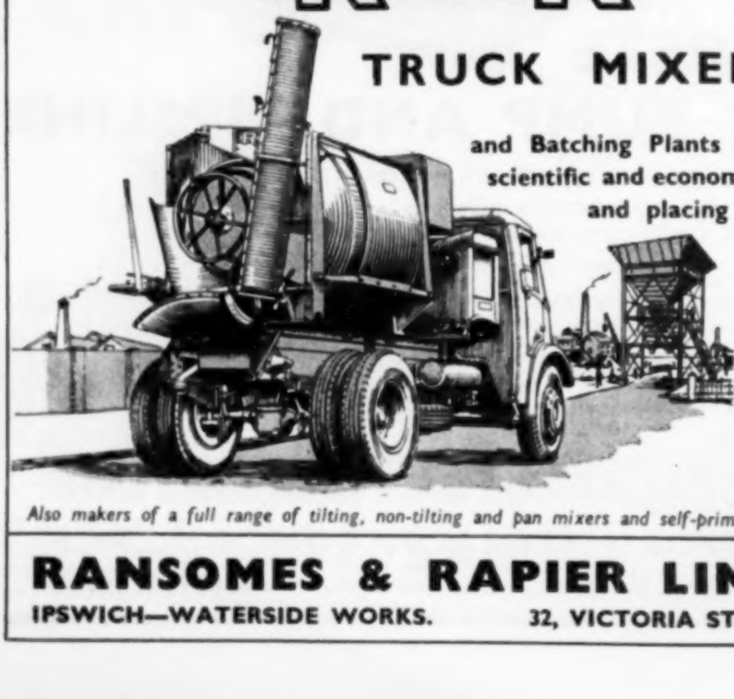

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CONCRETE AND CONSTRUCTIONAL ENGINEERING

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EDITORIAL NOTES

Concrete as a Barrier for Radiation.

THE physical properties and structural stability of concrete have been found to provide an effective barrier against the injurious effects and dissipation of radiation associated with radio-active substances and nuclear fission. The use of concrete as a barrier for X-rays is common and its use as a barrier for electro-magnetic waves of shorter wave-lengths is the subject of some recent research. X-rays (or Röntgen rays) and gamma-rays pass through concrete, but by making the concrete barrier thick enough the degree of absorption required can generally be obtained.

An effective barrier against X-rays, which are produced when cathode rays (a stream of negatively-charged electrons) strike a material object, may be produced by the intervention of materials of great mass of which lead, which was at one time generally used, and concrete are two examples. Many years ago plasters and concretes containing barytes (barium sulphate) were found to be effective substitutes for, and less costly than, lead, although for 100-kv. radiation the thickness of a barytes-concrete barrier is about five times that of a lead barrier of the same screening value, and ten to fifteen times for 200-kv. radiation. Ordinary concrete 4 in. thick is as effective against 400-kv. radiation as 0·05 in. of lead, and 20 in. of concrete is equivalent to about ½ in. of lead. For barriers for X-rays the value of unit thickness is greater the greater the penetrating power, that is the thickness of concrete required for 400 kv. is only slightly greater than that required for 200 kv. In one installation concrete walls 40 in. thick screen an apparatus emitting 2000 kv. The denser the material of which the barrier is composed, the greater is the protection afforded by unit thickness, and that is one reason why so much more concrete is required than lead, the specific gravities of the two materials being about 2½ and 11½ respectively. The protective qualities of concrete do not depend so much on the nature as on the mass of material. Because barytes concrete has a specific gravity of about 3½ it is therefore better for this purpose than ordinary concrete. The resistance of concrete to the penetration of X-rays is said to be due to the scattering of the rays before any great penetration is effected and, since the scattered waves are longer than the rays emitted by the original source, they are more readily absorbed by the remaining thickness of the concrete barrier.

Concrete is being used for the screening of apparatus where waves shorter than X-rays are produced. Radio-activity, the natural disintegration of atoms

of certain elements of high atomic weight, results in another atom of lower atomic weight and the emission of alpha-rays (a stream of positively-charged helium atoms of low penetrative power), beta-rays (a stream of negatively-charged electrons), and gamma-rays, which have a very short wave-length that is shorter than that of X-rays. As with X-rays, an effective screen against gamma-rays is one that permits the passage of so little radiation that it is harmless to persons and apparatus outside the screen. In artificial nuclear fission the screen must be effective against gamma-rays and neutrons. For gamma-rays alone, the screening medium must have great density or a high atomic number; lead and concrete are therefore suitable. For neutrons the medium must have a high hydrogen content, and therefore lead is valueless for this purpose. Ordinary concrete is therefore used for screening low-power nuclear-fission apparatus and a heavier concrete, such as barytes concrete, for apparatus of higher power. Concrete walls 5 ft. or more thick are generally required to prevent the passage of, say, 99 per cent. of the injurious rays.

Hitherto lead has been mainly recommended for the protective barriers against gamma-rays such as those emanating from radium, although for protection against high-power X-rays it is often more economical to use concrete barriers. It has been suggested that concrete might also be used for protection against gamma-rays if the necessary data regarding its screening value were available, and the results of research in America are reported in a recent number of the Journal of Research of the National Bureau of Standards. According to this report, which deals with radiation up to 3000 kv., the gamma-rays are reduced, or attenuated, within the concrete barrier by absorption, scattering, and other means, so that there is little probability of re-radiation of the high-speed electrons produced within the barrier. Close to the outer face of the barrier the intensity of the radiation (photons) escaping through the barrier may be appreciable, but it may be considerably reduced a short distance from the barrier. Thus the distance from the barrier to people outside must be taken into account as well as the distance of human beings from the source of the radiation. The data given assume that the permissible rate at which people working near the radio-active apparatus may absorb radiation is 0·3 röntgen per 48-hour working week, and if they are 7½ in. from the outer face of the barrier and about 3 ft. from the source the thickness of concrete required is about 20 in. if the strength of the radium source is 1000 millicuries (mc.), 11 in. for 100 mc., 8 in. for 50 mc., and 1½ in. for 10 mc. If the source is 10 ft. from human beings the thicknesses of concrete are 11½ in. for 1000 mc. and 2 in. for 100 mc.; apparently no barrier is required for this condition if the strength of the source is 50 mc. or less. The corresponding thickness of barriers of lead are about 10 per cent. to 15 per cent. of the thickness of concrete. If human beings are more than 3 ft. from the barrier the thickness of the concrete can be reduced by 1 in. (or lead by 0·15 in.).

Since the weight of a concrete barrier is greater than that of one of lead of the same screening value, larger foundations are required and the use of concrete for portable apparatus is precluded. A factor in favour of concrete as a screen against radiation is that other materials such as lead require structural support whereas a concrete barrier is self-supporting.

Construction with Moving Forms.—I.

By L. E. HUNTER, M.Sc., A.M.Inst.C.E.

[This is the first of a series of articles in which construction with moving forms will be discussed in detail, and which will illustrate with drawings and photographs successful methods of which the author has had experience.]

THE use of continuously-moving forms is an economical method of constructing a tall reinforced concrete structure which has more or less the same shape in plan throughout its height. In ordinary construction with panels of fixed wooden or steel shuttering, the shutters have to be removed, raised, and re-fixed for each lift of concrete. In moving-form construction a belt of forms, generally of wood and about 3 ft. to 4 ft. deep, is constructed on the ground and provides the shuttering for the faces of external and internal walls, columns, and other vertical surfaces. As the concrete is deposited, the forms are slowly and continuously

Fig. 1.—An Example of Moving-form Construction.

raised, generally by screw-jacks, until the top of the structure is reached. A typical arrangement of part of the equipment for moving-form shuttering is shown in *Fig.* 1. The jacks, which bite on vertical steel rods projecting from the foundation or the part of the walls already concreted, are fixed in timber yokes from which the forms are suspended. The forms comprise horizontal ribs to which vertical boards or steel plates are fixed. A deck, or working platform, is provided at the level of the top of the forms from which concrete is placed, reinforcement fixed, and the jacks operated.

Advantages of moving-form construction include the provision of a jointless structure (construction joints being a potential source of weakness), a saving in timber compared with fixed shuttering sufficient for two lifts of concrete, and a high salvage value of the yokes, ribs, lagging, and deck boards which can be used for similar structures. Scaffolding is unnecessary. The time spent in concreting is reduced compared with construction with fixed shutters but if, as described later, more than the ordinary amount of cover of concrete over the reinforcement is provided or the wall is made thicker for other reasons, additional

concrete may be required. The use of moving forms allows construction to proceed about four times as fast as with fixed shutters. The time taken in the preparation of moving forms is considerable, since the forms must be very accurately made, and the workmanship must be good, but this work can be done when the foundations are being prepared and in many cases provides employment in bad weather. The additional cost of night-work and other special conditions is generally more than off-set by other savings. The continuity of operations until concreting is completed, the centralisation of the concrete-mixing plant, and the exceptionally easy access to the work by means of the deck, tend to produce greater working efficiency than is the case with intermittent working with fixed shutters.

The finished external surface has a better appearance than that produced by fixed shutters, since there are no horizontal joints. However much rubbing-down is done to concrete after it has hardened, it is not possible to hide joint marks completely.

Generally, the structure must be not less than, say, 50 ft. high if full advantage is to be taken of moving-form construction. If the height is less than 50 ft. the advantage compared with fixed shutters is not sufficient to warrant the extra care and supervision required. The higher the structure the more economical are moving forms. Structures generally erected by moving forms are bunkers and silos comprising rectangular, square, circular, or polygonal bins or compartments. Moving forms are also used for other structures such as plain open structures with and without internal columns, plain buildings with the upper part set back, and cylindrical tanks. The problems in each of these types of structures are considered later.

Essential Factors.

The essential factors in moving-form construction are described in the following.

All parts of the forms must move upwards at the same rate and there must be no dragging of one section. For this requirement to be fulfilled, the forms must be accurately made. Dragging of one part may cause jamming which results in the work being stopped while the defective part is released.

Lateral support for the forms must be provided. In the case of long bins, the forms for the longer walls require considerable bracing.

Walls should be not less than 6 in. thick, because the depth of wet concrete in the form at any time is limited and if the weight of concrete is insufficient the concrete may be lifted as the forms move upwards. It is usual to deposit about 9 in. of concrete at each filling to keep the head of wet concrete constant throughout the walls. The friction of the forms is greatest when a thin skin of set cement adheres to the face of the forms, that is usually during the first day in use. Lifting of concrete that has initially set can occur easily in such circumstances unless the weight of the concrete is sufficient to withstand the dragging effect. From observations made by the writer, the coefficient of friction when sliding is just about to occur is about 0·25 to 0·3. It appears that the slower the rate of vertical movement the greater is the friction between the newly-placed concrete and the cement skin on the forms.

If moving-form construction is to be used, it is necessary to reduce extraneous

features and fittings as much as possible, that is, the faces of the wall should be plane throughout the height of the structure. Changes in thickness, either by set-backs or by tapering, can be allowed for if they are necessary. Vertical pipes should be inside the building. Projections such as canopies and galleries should be constructed after the moving-form construction is completed. In these cases the reinforcement connecting the projection to the main wall must be built into the walls, the projecting bars being laid against the face of the wall while the forms slide past, and afterwards bent down to the position required in the projecting feature.

Alterations to the design necessitating substantial alteration of the forms during their upward progress should be avoided. As described later, it is possible for considerable alterations to be made to the forms while the work is in progress, but such alterations must be known in advance so that allowance can be made for them when the forms are being made.

Moving-form construction must be continuous. From the time the first batch of concrete is placed the process must be carried through with only short breaks for meals, and concreting must not be otherwise stopped until the forms reach the top of the structure. It is sometimes possible even to avoid breaks for meals by staggering meal-times. Moving-form construction must not be stopped at night ; it is easy to imagine the difficulty of restarting the movement if the forms have been stopped until the concrete has hardened.

The arrangement of the reinforcement is an important factor and is different from that common if " lift " shutters are used, and therefore the designer must arrange the reinforcement to suit moving-form construction. Some designers prefer to provide only horizontal bars in the walls, and the writer considers that this arrangement has no ill effect on the strength of the structure. The continuous placing of the concrete with the accompanying uniform setting allows shrinkage of the concrete to occur at a more gradual and steady rate without resistance by the structure, and for this reason shrinkage cracks are uncommon in walls built by moving-form construction. It is advantageous to provide bars of not less than ½ in. diameter. Smaller bars tend to sag if they are horizontal. Bars without hooks are an advantage since, with horizontal bars only, the bars are not wired together but are fitted between vertical guides fixed to the forms. Numerous vertical bars can be inconvenient as they necessitate barrow-runs being in certain places, while the amount of work in steel fixing is increased, and this may result in a reduced rate of progress. If vertical reinforcement is necessary, it should be as little as possible. It is useful to have some means of seeing at a glance what is the correct spacing of the horizontal bars, and for this purpose vertical bars and column links are an advantage.

It is as easy in moving-form construction as in lifting-shutter construction for the mixture of the concrete to be varied, for example near the top of the structure the mixture may not need to be as rich in cement as at the bottom.

The supply of concrete materials must be carefully planned, since a lack of one of the materials stops the entire work. Not only must the materials be on the site in sufficient quantities, but the mixing plant must be capable of working for several days without a stop. Therefore, a stand-by plant of sufficient capacity is necessary. The hoists must have ample loading range so that if one breaks down the others can carry on without loss of progress.

Sufficient storage space must be available for not less than half the concreting materials and preferably all the reinforcement required. On congested sites this is often difficult to obtain, but with the use of ready-mixed concrete the need to store concreting materials is eliminated. However, with moving-form construction it is essential to be able to alter the mixture at short notice, and for this reason ready-mixed concrete is not so adaptable.

Concrete.

The quality of concrete is inevitably largely in the hands of the workmen making and placing it. Much depends on the skill and diligence of the supervisors but, even when this is of the very best, poor workmanship can and does occur since no foreman or resident engineer can be everywhere at the same time. If the workmen are conscientious there will be little trouble in the future; if they are incompetent, careless, or lazy, the results will appear as defects either immediately or in the not distant future. It is difficult, except at prohibitive cost, to make uniformly good concrete under site conditions because of variations in materials and workmanship, and the best that can be done is to reduce the effect of the uncertainty of workmanship as much as possible. This can be done as follows.

(1) By controlling the proportioning and mixing so that, when a mixture has been agreed upon, it cannot be altered without the consent of the engineer. Modern methods of mixing tend towards this end, but there is still scope for improvement. Truck-mixers are useful, but have their limitations. Weigh-batchers are a step in the right direction and give more effective control of the bulking of sand due to moisture content.

(2) By control of the water content; here, again, well-known principles are not always properly applied.

(3) By vibrating the concrete to a denser mass. Vibration has been proved to be useful, but is still sometimes looked upon with doubt.

(4) By "vacuum" concrete. This is in its initial stages in the United States of America, but may have wide use before it, especially for marine structures; it may reduce the amount of concrete cover required, and at the same time produce a sound and denser concrete than other methods now in use.

(5) By proper curing. Often too little attention is paid to curing.

(6) By providing a suitable cover of concrete to the reinforcement, taking into account the amount of protection required and the difficulties of fixing reinforcement.

(7) By appointing supervising engineers who understand the making and placing of concrete as well as design. Designers without experience of site work are generally too rigid in attempts to enforce the letter of a specification and do not know when to deviate from a specification for the betterment of the construction.

(8) By making joints that are clean, hacked, and suitably coated with grout. These requirements are essential for all constructional work, but frequently they are omitted and bad joints result.

Jointless Construction.

Moving-form construction is not only a method of shuttering. It is a different method of construction which comprises most of the well-known principles of

concrete construction by other methods, but ensures a structure the walls and columns of which are free from construction joints. Ordinary construction invariably consists of masses of concrete, each of which is expected to be joined to its neighbours to form a monolithic structure. No one can guarantee the effectiveness of a joint, and even if the joints are made in accordance with the best practice it is no guarantee that the work is going to be watertight. It is possible for pieces of timber, cigarette ends, paper, shavings, and the like to be inadvertently left in the joint. The only way to avoid defects due to poor joints is therefore to avoid joints in the structure. A structure built without joints and with reasonable care has no chinks for the weather to get in. Tall concrete structures, especially those in very exposed positions, are subjected to continual changes of temperature, wind pressure, and rain in varying degrees throughout their life, with the result that deterioration may set in, rust stains appear on the outer walls, and dampness slowly seep through defective parts of the walls. The repair of such a building is usually costly, as it is seldom noticed that the walls are leaking until they are in an advanced state of disrepair. The use of moving forms obviates the necessity for joints, and by this method the walls of structures of any size can be built without joints and consequently without defects due to bad joints. The writer has been engaged on large structures constructed by this method and has never noticed defects of a serious nature, nor has he heard of them occurring.

(*To be continued.*)

Impact of Boats against Piers.

Tests were made last year to determine the force with which a paddle steamer of 525 tons displacement hit a pier when berthing at different velocities of approach. The boat was moored against the reinforced concrete pier at New Holland, Lincolnshire (the extension of this pier is described in this journal for October, 1949), and was allowed to swing in alongside the pier in the usual manner of berthing. The blow was taken on a spring buffer which was calibrated so that measured compressions could be converted into the equivalent forces. Measurements were made of thirteen collisions with velocities, over the last 4 ft. 7 in. of travel, from 0·87 ft. to 1·24 ft. per second. The corresponding impact loads were 17·5 tons to 41·0 tons. The mean velocity was 1·05 ft. per second, and the mean load 27·9 tons. Although these results give information on the loads that may be expected on this particular pier, it is stated in the report of the test in "The Dock and Harbour Authority" that it is not possible to derive therefrom data for general application. The tests were made under the direction of the Civil Engineer of the Eastern Region of British Railways.

Horizontal Forces exerted by Crowds.

In a recent number of the "Surveyor," Mr. L. R. Creasy describes some tests made to determine the horizontal force exerted by a crowd of people pushing against a barrier. From the results of the tests, the author recommends that in the design of barriers the horizontal load per foot length of the barrier should be assumed to be 15 lb. per foot depth of the crowd. Therefore a barrier behind which a crowd 10 ft. deep can accumulate should be designed for a horizontal load of $15 \times 10 = 150$ lb. per foot of length.

Book Reviews.

"Engineering Structures." (London: Butterworth's Scientific Publications, Ltd. 1949. Price 25s.)

In this book, which is volume two of the papers of the Colston Research Society, are printed fifteen of the contributions to the symposium held by the Society in September last. The subject common to all the papers is non-linearity in the behaviour of structures, which in some cases arises from plasticity or creep of the materials. In other cases it may be due to large deflections which may develop gradually or suddenly. The term "structures" is interpreted widely so as to include problems associated with buildings, ships, and aeroplanes. Therefore some of the subjects relate mainly to metallic construction. Apart from the general theoretical papers, one of interest to concrete engineers is that by Mr. F. Vogt in which he deals analytically with the effect of shrinkage on the deformation of concrete subjected to a sustained load.

"Der Stahlbetonbau." By C. Kersten. Vol. III. Ninth edition. (Berlin: Wilhelm Ernst & Sohn. 1949. Price 8.20 DM.)

The third volume of Professor Kersten's book on reinforced concrete design gives over a hundred examples of many types of buildings illustrating the application of the theory described in the preceding volumes. In addition to the more common reinforced concrete members, the examples include precast members and prestressed beams. Clapeyron's analytical and Ritter's graphical methods of solving continuous beam problems are described.

"Lexique Technique: Français-Anglais et Anglais-Français." (Paris: Institut Technique du Batiment et des Travaux Publics. 1950. Price 700 francs.)

This small technical dictionary contains nearly 5,000 entries in the French-English part, over 7,000 entries in the English-French part, and thirty-five pages of tables for converting metric measures to British measures and vice versa. The subtitle indicates that the terms interpreted concern materials for public works, but the publisher's statement that the book is a glossary of French, English, and American technical terms relating to engineering equipment is a better description. The glossary is apparently intended for French engineers when ordering British or American mechanical equipment. Omissions are inevitable in a small book. Although concrete, cement, and reinforcement are dealt with fairly well, shuttering and synonyms thereof are omitted with the exception of "mould", which in some places is spelled "mold". There is some confusion between quick setting and rapid hardening as applied to cement, and between stress and strain. There appear to be several instances of lack of co-ordination between the two parts; for example "acier doux" is given as "soft steel" in the French-English part, but the true meaning, "mild steel," is given in the English-French part. In some cases American and British terms are given where these differ.

"Navrhování Betonových Konstrukcí Podle Stupně Bezpečnosti." By K. Hruban. (Brno: Rovnost. 1949. Price Kčs. 138.)

The new method of designing reinforced concrete structures used in the latest Czechoslovakian regulations is explained in detail, and a summary in English is given. The plastic theory on which the method is based is described by the author in this journal for December 1949. Ordinary bending, direct compression, bending combined with compression, direct tension, and bending combined with twisting are considered in the book.

"L'Astr Solidale." By A. Linari. Vol. 1. (Naples: B. Pellerano Del Gaudio. 1950. Price 700 lira.)

In this first volume of a series of books in the Italian language dealing with continuous-beam structures, the author considers beams of one span, beams continuous over several spans, and some types of frames. The method of solution, which is based on the position of "nodes," or fixed points, and elastic weights, differs from some others insofar that an increase in the number of members does not detract from the simplicity of the calculations, which depend on knowing the value of one unknown quantity determined by the aid of tables. When the value of this factor is known it is possible to calculate directly the bending moments and rotation at the ends of each span and the shearing force and deflection at any section. Expressions for influence lines are also readily derived. Elastic movement of the joints can be taken into account. A few numerical examples are given which help to elucidate the method.

Prismatic Structures with Transverse Stiffeners.

By Professor H. CRAEMER (ALEXANDRIA).

In the first of the articles on prismatic structures by Mr. A. J. Ashdown in this journal for October, 1948, reference is made to the writer's analysis of such structures, and the essentials of this analysis should be understood before the theory of prismatic structures with transverse stiffeners is considered.

The General Theory.

The edges of plane slabs forming the simplest prismatic structures are parallel. Statically-determinate prismatic structures are supported in two planes (generally at right-angles to the edges) or are fixed at one end. Since each slab generally offers much less resistance to displacement normal to its plane than to displacement in its plane, the latter displacements can be neglected. The structure is therefore a series of continuous slabs each supported on the edges of adjacent slabs.

Consider the equilibrium of the edges of the slabs in *Fig. 1a*. The shearing forces per unit width of slab on each side of T are s_{TS} and s_{TU} as in *Fig. 1b*, the moments m_T acting as shown. Equilibrium can exist only if axial forces N_{TS} and N_{TU} act, and it can therefore be deduced that

$$\left.\begin{matrix} N_{TS} \\ N_{TU} \end{matrix}\right\rangle 0{\cdot}5[s_{TS}(\pm \tan\theta_T + \cot\theta_T) + s_{TU}(\pm \tan\theta_T + \cot\theta_T)].$$

Any slab TU (*Fig. 1c*) is subjected to an axial force P_{TU}, where

$$P_{TU} = N_{UT} - N_{TU},$$

and is subjected to a moment M_{TU} acting as in *Fig. 1d*. At the edge T of slab TU, the moments produce the stresses $f'_{TU} = -f'_T$, and $f'_{UT} = f'_T$, where $f'_T = \dfrac{M_{TU}}{z_{TU}}$;

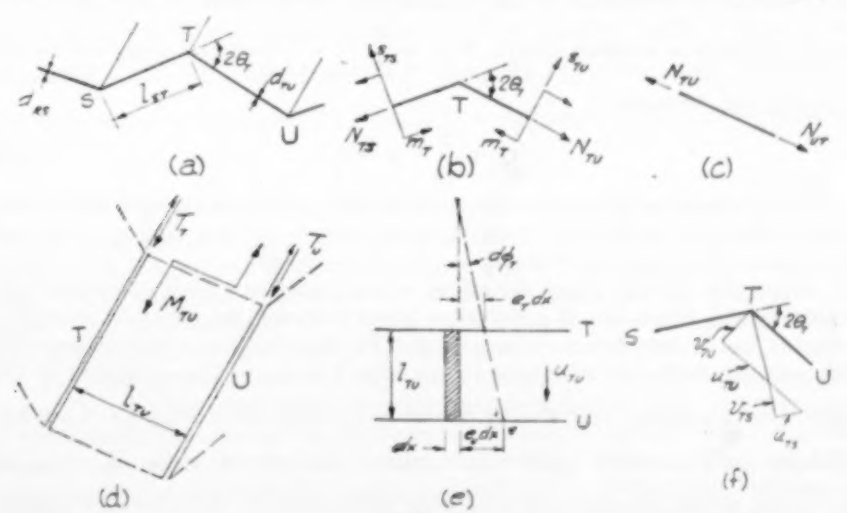

Fig. 1.

z_{TU} is the section modulus, that is $\frac{d_{TU}l_{TU}^2}{6}$, in which d_{TU} is the thickness of the slab TU. It is assumed that Hooke's law applies and that tensile stresses are positive. The stresses f' are different on either side of an edge. Balance is effected by the shearing forces T_T, T_U, etc. (*Fig.* 1*d*) which act along the edges. The forces T produce additional bending in the slabs, which induces the stresses

$$f_T = -f'_T + \frac{2}{A_{TU}}(2T_T + T_U) \quad . \quad . \quad . \quad . \quad (1a)$$

and

$$f_U = f'_T - \frac{2}{A_{TU}}(T_T + 2T_U) \quad . \quad . \quad . \quad . \quad (1b)$$

where A_{TU} is the cross-sectional area of slab TU($= d_{TU}l_{TU}$). Solving for T,

$$T_T = \frac{A_{TU}}{6}(f'_T + 2f_T + f_U) \quad . \quad . \quad . \quad . \quad (2a)$$

and

$$T_U = \frac{A_{TU}}{6}(f'_T - f_T - 2f_U) \quad . \quad . \quad . \quad . \quad (2b)$$

The equation similar to (1*b*) but applicable to slab ST is another expression for f_T and, by equating to (1*a*), an equation of three shearing forces is obtained :

$$\frac{T_S}{A_{ST}} + 2T_T\left(\frac{1}{A_{ST}} + \frac{1}{A_{TU}}\right) + \frac{T_U}{A_{TU}} = \frac{f'_S + f'_T}{2} \quad . \quad . \quad . \quad (3)$$

By transposition of (2*b*) and equating to (2*a*),

$$A_{ST}f_S + 2f_T(A_{ST} + A_{TU}) + A_{TU}f_U = A_{ST}f'_S - A_{TU}f'_T \quad . \quad . \quad (4)$$

If $A_{ST} = A_{TU} = A$, equation (4) becomes

$$f_S + 4f_T + f_U = f'_S - f'_T \quad . \quad . \quad . \quad . \quad (4a)$$

Since these are similar to the equations of the theorem of three moments, similar methods of solution apply. If $r_S = \frac{f_S}{f_T}$, $r_T = \frac{f_T}{f_U}$, etc., for a bay not loaded, equation (4*a*) becomes

$$r_S + \frac{1}{r_T} + 4 = 0 \quad . \quad . \quad . \quad . \quad . \quad (4b)$$

In a succession of several non-loaded slabs, r is constant and is $-0{\cdot}268$. If slab TU only is loaded, from equation (4*a*), $rf_T + 4f_T + f_U = -f'_T$ and $f_T + 4f_U + rf_U = f'_T$; therefore $f_T = -f_U = -0{\cdot}366f'_T$.

When the stresses along the edges of the slabs have been calculated, the displacements, which are of importance when deducing the effect of transverse stiffeners, can be determined. Consider slab TU of which the angular deformation is ϕ_T and the displacement normal to the edge T is u_{TU}. The elongation of the edges of an elementary strip dx (*Fig.* 1*e*) are $e_T\,dx$ and $e_U\,dx$, e being $\frac{f}{E}$. Therefore adjoining sections rotate against one another through an angle $d\phi_T$ equal to $\frac{e_U - e_T}{l_{TU}}dx$, which expression is called the differential of the "area of angular

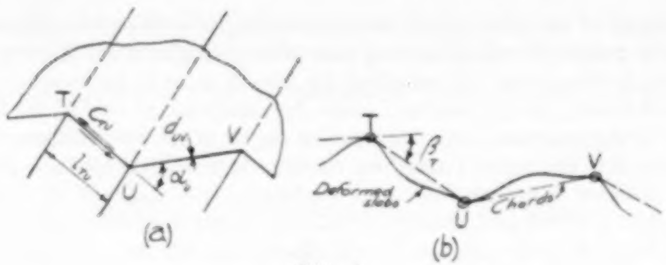

Fig. 2.

rotation" and is a more general case of the relation $d\phi = \frac{M}{EI}dx$ applicable to the angular deformation of beams. In accordance with Mohr's theorems, it can be stated that the rotation ϕ and the displacement u of any section are equal to the shearing force S and the bending moment M respectively if the "area of angular rotation" is considered to be the load on the slab.

In applying this rule the end conditions must be taken into account. If two sections are supported immovably, $M = 0$. If one end is fixed against rotation and displacement, $S = 0$ and $M = 0$.

To maintain continuity of the adjacent slabs ST and TU at the joint T, displacements v_{TU} and v_{TS} must occur, and these are considered to be positive if acting as in *Fig. 1f*. It can be shown that

$$\left.\begin{matrix} v_{TS} \\ v_{TU} \end{matrix}\right\rangle \tfrac{1}{2}[u_{TS}(\pm \tan\theta_T - \cot\theta_T) + u_{TU}(\pm \tan\theta_T + \cot\theta_T)].$$

The stresses normal to the plane of the slab produced by the curvature are negligible. The displacement of the edge is clearly determined by u_{ST} and v_{TS}, or u_{TU} and v_{TU}.

The only modification to the foregoing analysis when considering statically-indeterminate structures is to take account of the indeterminacy in M_{TU} in the expression $f'_T = \frac{M_{TU}}{z_{TU}}$; the values of f'_T obtained therefrom, when substituted in equations (3) and (4), give the stresses in a statically-indeterminate prismatic structure.

The Effect of Transverse Stiffeners in Closed Prismatic Structures.

In the following analysis it is assumed that the stiffener is connected only to the slabs forming the prismatic structure but is not supported in any other way. To analyse the stresses for this condition, some simplification must be made. The deformation of the stiffener, which may be a wall, floor, rib, or the like, is negligible compared with the displacement of the slabs.

The effect of the stiffener is to produce forces C_{TU} (*Fig. 2a*) which must be in equilibrium with other forces and moments to which the slabs are subjected. It is evident that the forces C must be zero in a three-sided open or closed prismatic structure, otherwise the stiffeners would not be in equilibrium. The same condition applies to a rectangular structure subjected to symmetrical loading.

Consider a closed prismatic structure of n sides, and therefore n edges. If

l be the length of the sides and α the exterior angle between two adjacent sides, the shape is defined if n sides l_1 to l_n and $(n - 3)$ angles α are known, although if the shape is triangular it is sufficient for $n(= 3)$ sides to be known. The load and the n unknown (as yet) forces C cause deformations as in *Fig. 2b*, the length of the side being unaltered, and the exterior angles at the intersection of adjacent chords being β. The figure formed by the chords is defined by $(n - 3)$ angles β. Since the stiffener is assumed not to be deformed, the angles β are equal to the original angles α, which fact, combined with the ordinary conditions of equilibrium, enables n forces C to be calculated. With several stiffeners the number of unknown forces C is accordingly increased, and the equation for each stiffener can be derived as for a single stiffener. If the structure and load are symmetrical, as in many cases, the number of unknowns is reduced. The example that follows explains the method.

Consider the octagonal shaft of the water tower (*Fig.* 3) subjected to wind pressure and suction, the values of p being the intensity of normal pressure. If the shaft is free for a height H and is unrestrained by stiffeners, the stresses

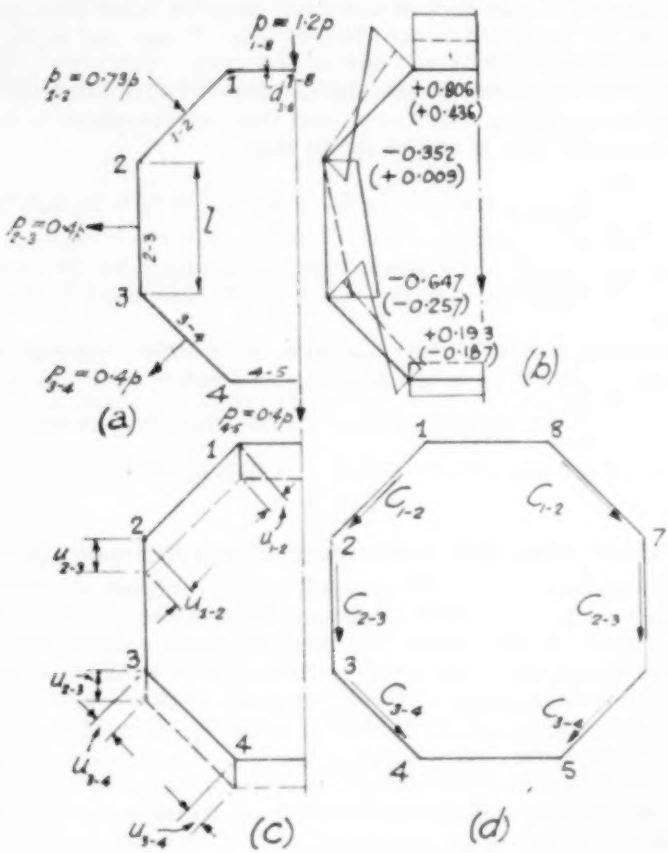

Fig. 3.

at the base of the tower vary as shown by the full lines in *Fig. 3b*. The magnitudes of the stresses f are given by the coefficients (not in brackets in *Fig. 3b*) multiplied by $\frac{pH^2}{ld}$, and are calculated from the equations for statically-determinate prismatic structures given in the first part of this article. The next step is to calculate the deflection u at the top of each side in the plane of the side (*Fig. 3c*). Since the stresses vary parabolically, the deflection u_{1-2} can be expressed as $\frac{f_1 - f_2}{lE} \cdot \frac{H^2}{4}$. Therefore the deflections at each intersection of the sides are given by expressions having the form $\gamma \frac{pH^2}{El^2d}$, γ being $+ 0{\cdot}290$ for u_{1-2}, $+ 0{\cdot}074$ for u_{2-3}, and $- 0{\cdot}210$ for u_{3-4}; $u_{8-1} = u_{4-5} = 0$.

If the shaft is stiffened at the top by a floor, some of the forces C (*Fig. 3d*) can be eliminated because of symmetry, and $C_{8-1} = C_{4-5} = 0$. For the three remaining forces C, equilibrium requires that the sum of the components in any direction shall be zero, that is $C_{2-3} + \frac{C_{1-2} + C_{3-4}}{\sqrt{2}} = 0$, or $C_{2-3} = -\frac{C_{1-2} + C_{3-4}}{\sqrt{2}}$. The next step is to calculate the stresses at the base of the tower due to C_{1-2}, C_{2-3}, and C_{3-4}, and by well-known methods, and eliminating C_{2-3}, these stresses are:

$$\left.\begin{aligned}
f_1'' &= (1{\cdot}91C_{1-2} + 0{\cdot}41C_{3-4})\frac{H}{dl^2} \\
f_2'' &= (-\,3{\cdot}55C_{1-2} - 2{\cdot}05C_{3-4})\frac{H}{dl^2} \\
f_3'' &= (2{\cdot}05C_{1-2} + 3{\cdot}55C_{3-4})\frac{H}{dl^2} \\
f_4'' &= (-\,0{\cdot}41C_{1-2} - 1{\cdot}91C_{3-4})\frac{H}{dl^2}
\end{aligned}\right\} \quad . \quad . \quad . \quad . \quad (6)$$

These stresses increase linearly from top to bottom of the tower, so that the corresponding deflections at the top of the shaft are given by equations of the form $u_{1-2}'' = (f_1'' - f_2'')\frac{H^2}{3lE}$.

Combination with the expression for f_1'', etc., gives

$$\left.\begin{aligned}
u_{1-2}'' &= (1{\cdot}82C_{1-2} + 0{\cdot}822C_{3-4})\frac{H^3}{Edl^3} \\
u_{2-3}'' &= -\,1{\cdot}87(C_{1-2} + C_{3-4})\frac{H^3}{Edl^3} \\
u_{3-4}'' &= (0{\cdot}822C_1 + 1{\cdot}82C_3)\frac{H^3}{Edl^3} \\
u_{4-5}'' &= u_{1-8}'' = 0.
\end{aligned}\right\} \quad . \quad . \quad . \quad . \quad (7)$$

Since the stiffening floor can be displaced only as a whole, the combined displacements due to wind pressure and stiffening are

$$u_{1-2} + u_{1-2}'' = u_{3-4} + u_{3-4}'' = \frac{u_{2-3} + u_{2-3}''}{\sqrt{2}}.$$

Substituting the values of u and u'' from equations (6) and (7) and reducing, $C_{1-2} = -0{\cdot}248pHl$, $C_{2-3} = -0{\cdot}004pHl$, and $C_{3-4} = 0{\cdot}253pHl$. Substituting in equations (6), and superimposing the stresses thus determined on those shown by the full lines in *Fig. 3b*, the stresses shown by the broken lines and the coefficients of $\frac{pH^2}{dl}$ in brackets in *Fig. 3b* are obtained, and are the stresses at the base of the stiffened tower. The large reduction of stresses and the almost linear variation of stress resulting from the provision of the floor should be noted. The effect is more pronounced if one or more intermediate floors are provided as is often the case in water towers.

Resistance of Compressed Concrete to Chemical Attack.

Some tests on the effect of magnesium sulphate on cement mortars in a state of strain are described in a recent number of "Il Cemento". The bending strengths of specimens of ordinary Portland cement concrete subjected to compression and bending while immersed in a 5 per cent. solution of magnesium sulphate and in fresh water were determined, and it was shown that there is a considerable increase in resistance to chemical attack if the mortar is in compression. On the other hand, if the concrete is in tension, the disintegrating effect of sulphate solutions is accelerated. The bending tests were made after periods of 14 days, and after 1, 3, 4, 5 and 6 months, on 1 : 3 mortar specimens 1⅛ in. square and 6¼ in. long. All specimens were cured in fresh water for 14 days and then immersed in the solution or in water. Some of the immersed specimens were not loaded and others were subjected to compression and bending in one plane by the apparatus shown in *Fig.* 1. The resistance to bending was determined for the unloaded specimens and for two conditions of the loaded specimens : (1) with the moment in the same sense as that when the specimen was immersed, and (2) with the moment in the opposite sense to that when immersed.

At the end of six months the strength of unloaded specimens in the solution was about 75 per cent. of the strength at 14 days, whereas the strength of the specimens in water, whether unloaded or tested in accordance with conditions (1)

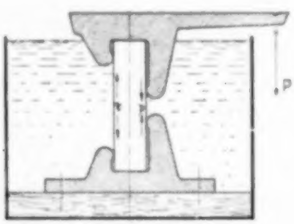

Fig. 1.

and (2), increased by about 20 per cent. The strength at six months of the specimens in the sulphate solution, when tested in accordance with condition (1), was about half the strength at 14 days, but was about one-third greater than the strength at 14 days when tested in accordance with condition (2), and about 15 per cent. greater than the strength at six months of the specimens immersed in water. This variation in resistance is of interest in connection with the use of prestressed concrete in marine structures.

Omnibus Station and Garage, Dublin.

THE reinforced concrete framework (*Fig.* 1) of the new omnibus station and offices for Coras Iompair Eireann (Irish Transport Company) at Store Street, Dublin, has recently been completed. This part of the structure was built in advance of the general building work. The site has the shape of a quarter of an ellipse. The main building fronts on to the two straight sides of the site, which are about 242 ft. and 200 ft. long respectively. Details of the structure are given in *Figs.* 2 and 3.

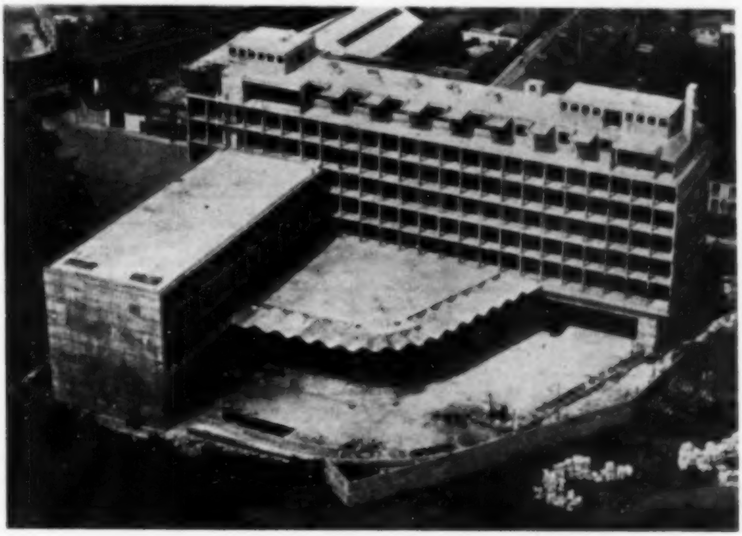

Fig. 1.—New Omnibus Station in Dublin.

The wings of the main building are 55 ft. wide. The north wing is 100 ft. high above street level and comprises seven stories and a mezzanine floor. The west wing is 58 ft. high and comprises four stories. Two rooms for ventilating plant and a chimney rise above the general level of the roof. A basement extends over the whole site. The part of the site which is not covered by the main building is to be used as an omnibus yard, and a covered concourse is provided in the angle between the two wings of the building.

The upper floors are designed for the superimposed loads specified for offices and restaurants in the British Standard Code, Chapter V. The open part of the ground floor over the basement is designed to carry omnibuses and pavings which are to be laid later. The external faces of the gable walls and all exposed concrete at and above the level of the first floor are to be faced with masonry. The building is carried on a raft foundation which forms the basement floor and, as the ground contains tidal water, the whole of the construction below ground is waterproofed with asphalt applied to the outer faces of the structural concrete. The basement drains are encased in concrete and are contained within the asphalt tank. The structural arrangement was devised to simplify the work of asphalting as much as possible.

The basement floor, which is generally 11 ft. below street level, is mainly an 18-in. reinforced concrete slab (*Fig.* 6), which is thickened under the columns. Where the floor is 16 ft. 6 in. and 19 ft. below street level, the slab is respectively 30 in. and 36 in. thick, the greater thickness being required to resist the greater water pressure and because of the absence of intermediate supports. *Fig.* 4 shows the basement and the commencement of

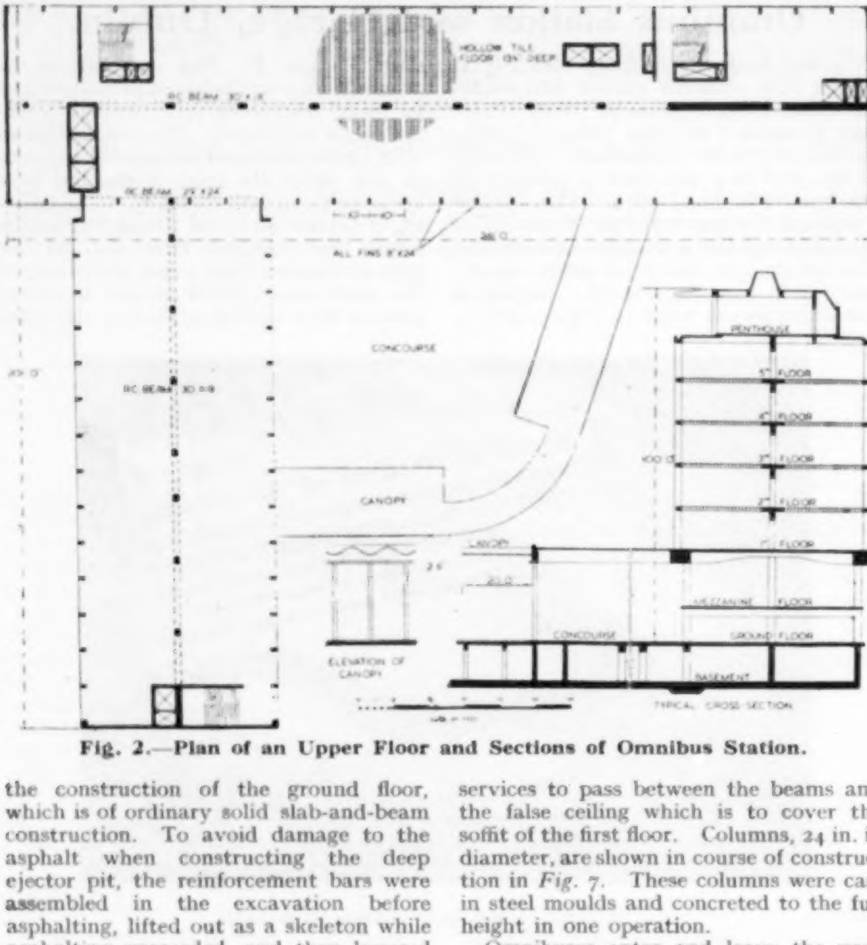

Fig. 2.—Plan of an Upper Floor and Sections of Omnibus Station.

the construction of the ground floor, which is of ordinary solid slab-and-beam construction. To avoid damage to the asphalt when constructing the deep ejector pit, the reinforcement bars were assembled in the excavation before asphalting, lifted out as a skeleton while asphalting proceeded, and then lowered into position for concreting (*Fig.* 5).

The columns between ground and first-floor levels are inset from the building line above the first floor, and the windows are outside the columns. The columns are spaced at 20-ft. centres, and the vertical fins above first-floor level are at 10-ft. centres and staggered in relation to the columns. The loads from the fins are transferred to the columns by an edge-beam 3 ft. 9 in. deep and 4 ft. 6 in. wide; the beam is subjected to bending and twisting. The cantilever bending moments at the columns are resisted by transverse beams, which are reduced in depth towards the middle row of columns to allow ventilating ducts and other services to pass between the beams and the false ceiling which is to cover the soffit of the first floor. Columns, 24 in. in diameter, are shown in course of construction in *Fig.* 7. These columns were cast in steel moulds and concreted to the full height in one operation.

Omnibuses enter and leave the yard through large openings under each wing of the building. The superstructure over each opening is carried on three portal frames, in five of which the beams are 9 ft. deep and 2 ft. and 2 ft. 6 in. wide. In the other frame, which is that under the centre of the north wing, a 24-in. wall between the first and second floors acts as the beam. Where the openings occur the first floor is of cellular construction, and a thin slab of reinforced concrete takes the place of the false ceiling provided elsewhere. *Fig.* 8 shows the construction of the beam of the portal frame over the exit; this beam has a clear span of about 56 ft.

The shape of the concourse roof in plan

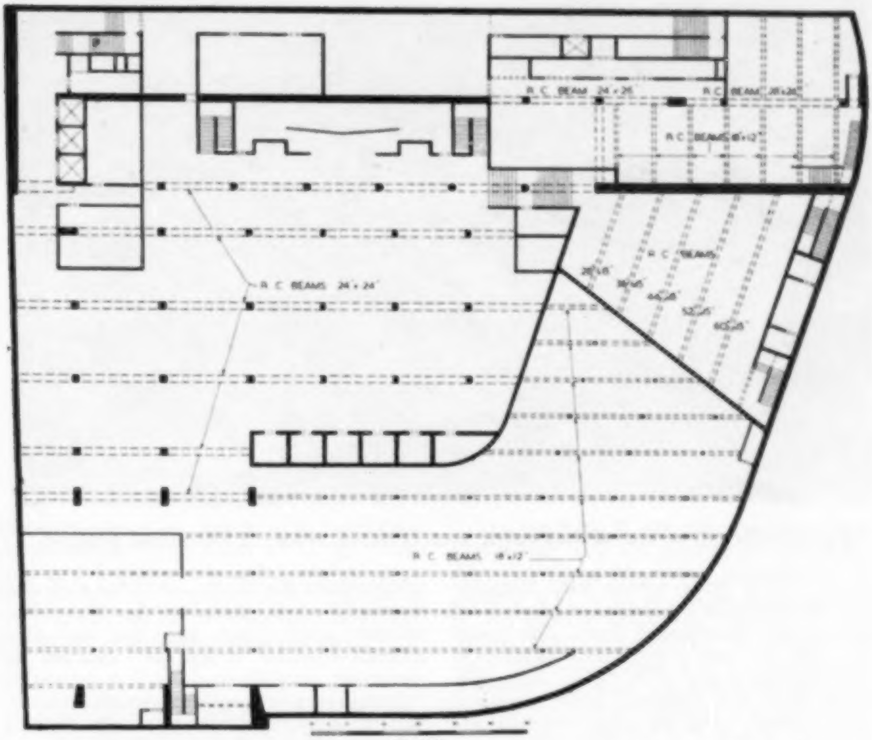

Fig. 3.—Plan of Basement of Omnibus Station, showing Ground-Floor Beams.

is determined by the shape of the site, uniform width being maintained for its full length. This concrete roof is supported by the main building on the two straight sides and by slender concrete columns, which will be behind a glass screen extending the full height, along the remainder of its periphery. The cantilevered canopy projecting over the yard overhangs 20 ft. The mezzanine floor under the front of the canopy is the control platform; it is cantilevered from a curved wall and will be later enclosed in glass above the parapet. The roof of the concourse comprises a 3-in. solid slab on two-way diagonal beams (*Figs.* 9 and 10). The average span is about 77 ft. in one direction and 80 ft. in the other; the overall depth of the construction is 3 ft. 9 in. The beams are arranged to intersect at the centres of the periphery columns which are spaced at 10-ft. The canopy is a corrugated slab 3 in. thick, the pitch of the corrugations being 10 ft. The valleys of the corrugations occur at the columns where the main beams of the roof meet; this results in a good structural arrangement and simplifies the placing of the reinforcement. *Figs.* 9 and 11 show two stages in the construction of the canopy. The triangular spaces between the intersecting roof beams are later filled with concrete to provide an anchorage for the tensile reinforcement of the canopy which projects into them. Hardboard, jointed in a regular pattern and supported by slats fixed to templates, was used to shutter the soffit of the canopy, and produced a smooth finish. In the curved part of the canopy the pitch of the corrugations was arranged to conform as nearly as possible to the pitch of 10 ft. along the outer edge, the effect of the curve being to fold up slightly the corrugations within the angle.

The upper floors of both wings are nearly identical in construction except for the recessed balcony on the third floor

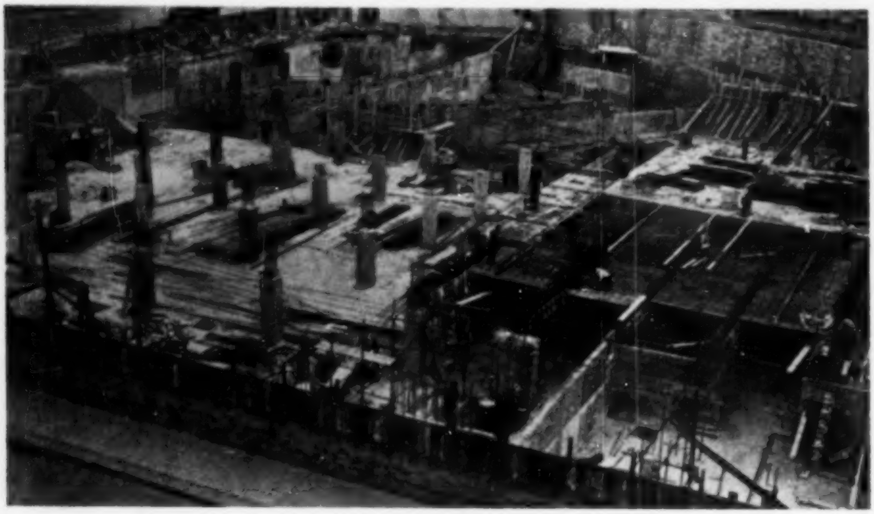

Fig. 4.—Construction of Basement and Ground Floor of Omnibus Station.

of the north wing where twin circular concrete columns are provided in place of each of the rectangular fins. The floors are $13\frac{1}{2}$-in. hollow-tile slabs supported on a central beam, 3 ft. $7\frac{1}{2}$ in. deep and 18 in. wide and spanning 20 ft. between the columns, and on edge ribs 21 in. wide within the thickness of the slab and spanning 10 ft. between the fins. The central beam occurs over one wall of a central corridor which will contain an overhead duct for services. A concealed lighting-duct is provided in each 10-ft. bay of floor; this arrangement allows a

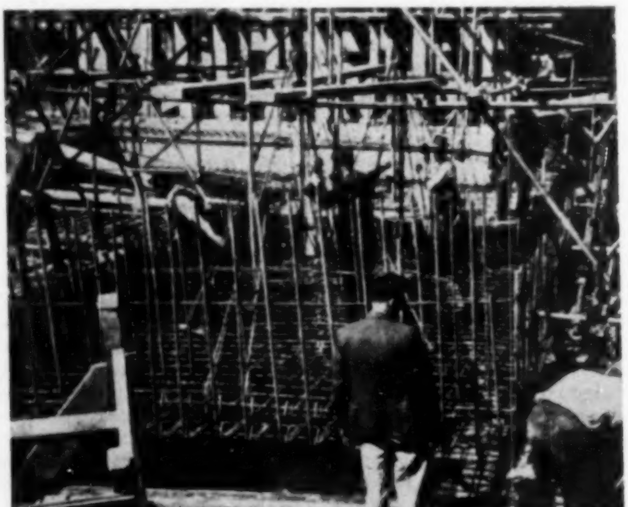

Fig. 5.—Lowering the Reinforcement for the Bottom of the Ejector Pit at Omnibus Station.

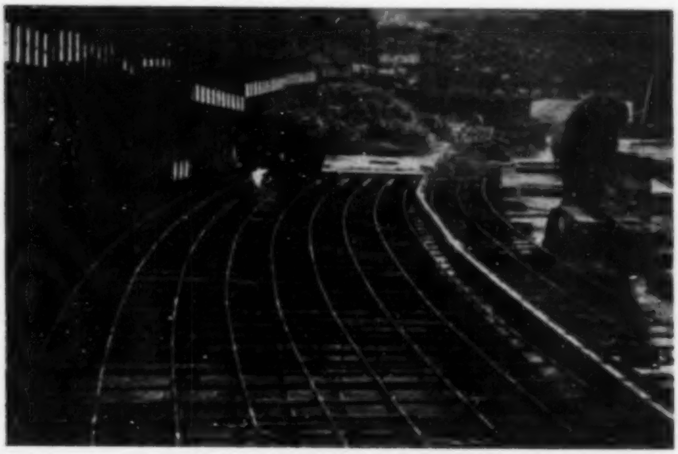

Fig. 6.—Reinforcement for Raft and Retaining Wall of Omnibus Station.

large measure of freedom in planning accommodation in units of 10 ft. The ribs in the hollow-tile slab are 6 in. wide and are at 1 ft. 10 in. centres. Two rows of 8-in. by 10-in. fireclay tiles on edge are provided between adjacent ribs.

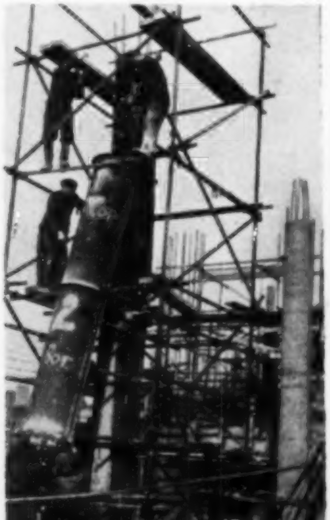

Fig. 7.—Construction of 24-in. Columns for Omnibus Station.

The roofs at fourth-floor level and the sixth floor, which is partly roof and partly floor, comprise transverse beams at 10 ft. centres and solid slabs because of concealed gutters along the edges. A false ceiling will be provided. The main roof at seventh-floor level is of beam-and-slab construction with cantilevered canopies on the south side and at the west end, where the canopy is T-shape on plan and is a double cantilever with the wings sloped slightly upwards towards the edges.

Concrete shafts are provided for the full height of the building for each of the three passenger lifts and a goods lift. The intake, supply, and exhaust-air shafts are entirely in concrete and also extend the full height of the building. The flue from the boilers is also constructed in concrete with firebrick lining and glass-wool insulation between the concrete and firebrick.

All openings and sleeves for fixing services were formed or built in the structural members as construction proceeded, but no holes were formed for fixing the masonry or windows as it was considered that the positions of such holes could not be determined with sufficient accuracy and would complicate the shuttering unnecessarily. The provision of these holes was taken into account when arranging the reinforcement.

With the exception of the 3-in. corrugated canopy, where the concrete mixture is $1 : 1\frac{1}{2} : 3$, all reinforced concrete is

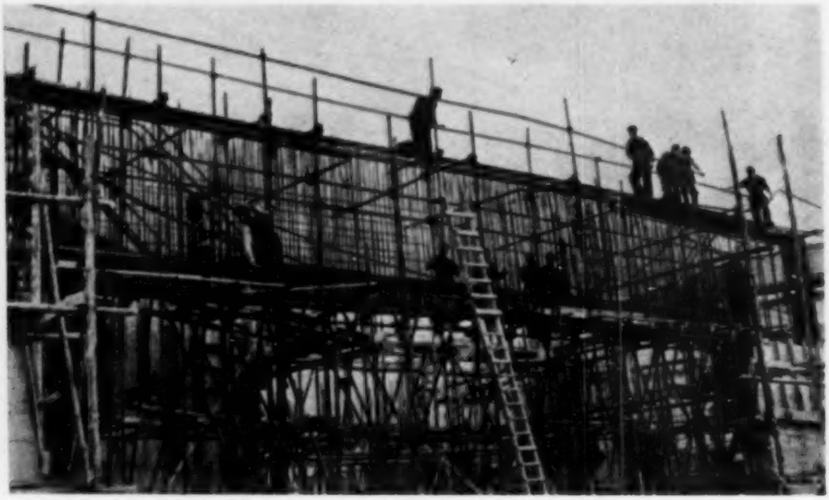

Fig. 8.—Reinforcement of Beam of Portal Frame at Omnibus Station.

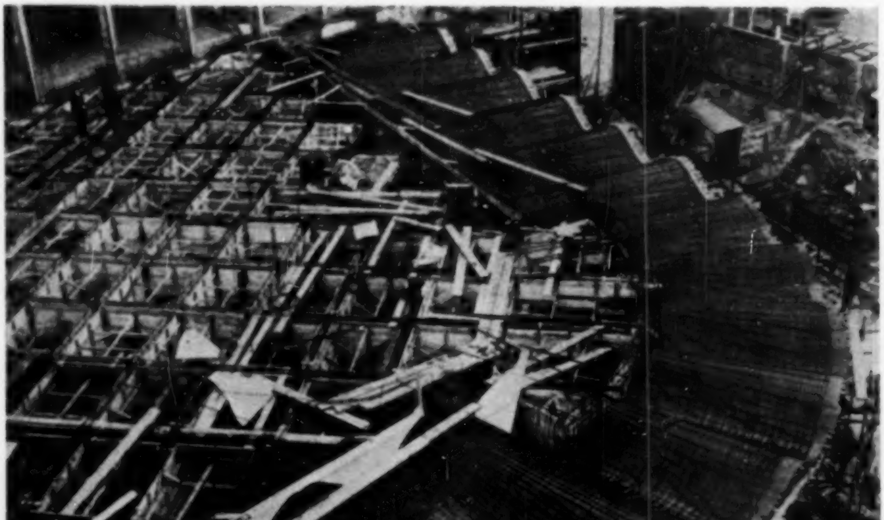

Fig. 9.—Concourse Roof of Omnibus Station and Canopy During Construction.

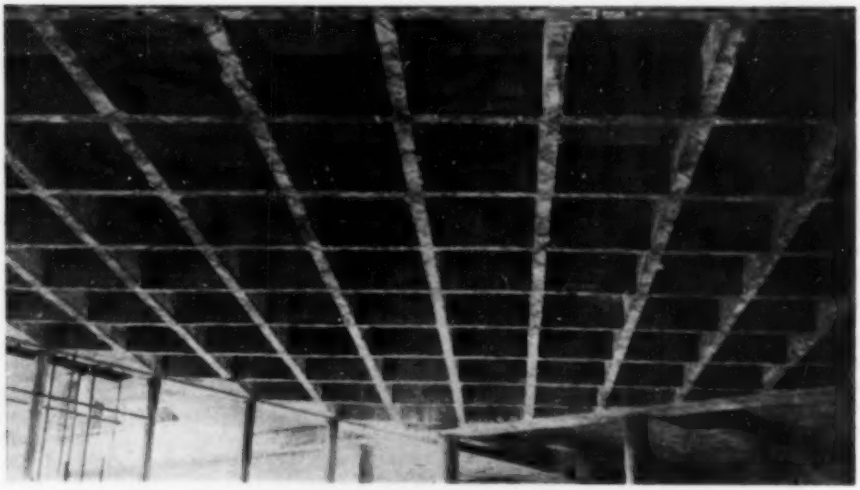

Fig. 10.—Underside of Concourse Roof of Omnibus Station.

1 : 2 : 4. The aggregate is washed sand and gravel. The average strengths of cubes of concrete at seven days were from 3500 lb. to 4500 lb. per square inch, the greatest strengths being about 6000 lb. per square inch. The steel reinforcement complied with British standards.

Mr. Michael Scott, F.R.I.A.I., is the architect for the omnibus station at Store Street, Dublin, the contractors being Messrs. John Sisk & Son (Dublin), Ltd.

Omnibus Garage at Donnybrook.

The first part of an additional omnibus garage at Donnybrook for Coras Iompair Eireann was also recently completed. The structure comprises 147 cast-in-situ piles supporting columns which carry a roof consisting of ten thin-slab vaults (*Figs.* 12

Fig. 11.—Detail of Canopy at Omnibus Station.

Fig. 12.—Garage at Donnybrook.

and 13). The enclosing walls and the permanent floor of the structure and reinforced concrete buildings for washing and servicing vehicles and for administration will be completed later.

The width of the structure is 110 ft. and the length about 400 ft. Each vault is 40 ft. wide and spans 104 ft. between main columns. The height to the underside of the longitudinal beams is 20 ft. The roof is in two parts each comprising five vaults and separated by an expansion joint at which twin columns and twin beams are provided. A feature of the roof is that top lights 7 ft. 6 in. wide extend the full length of the vaults. The

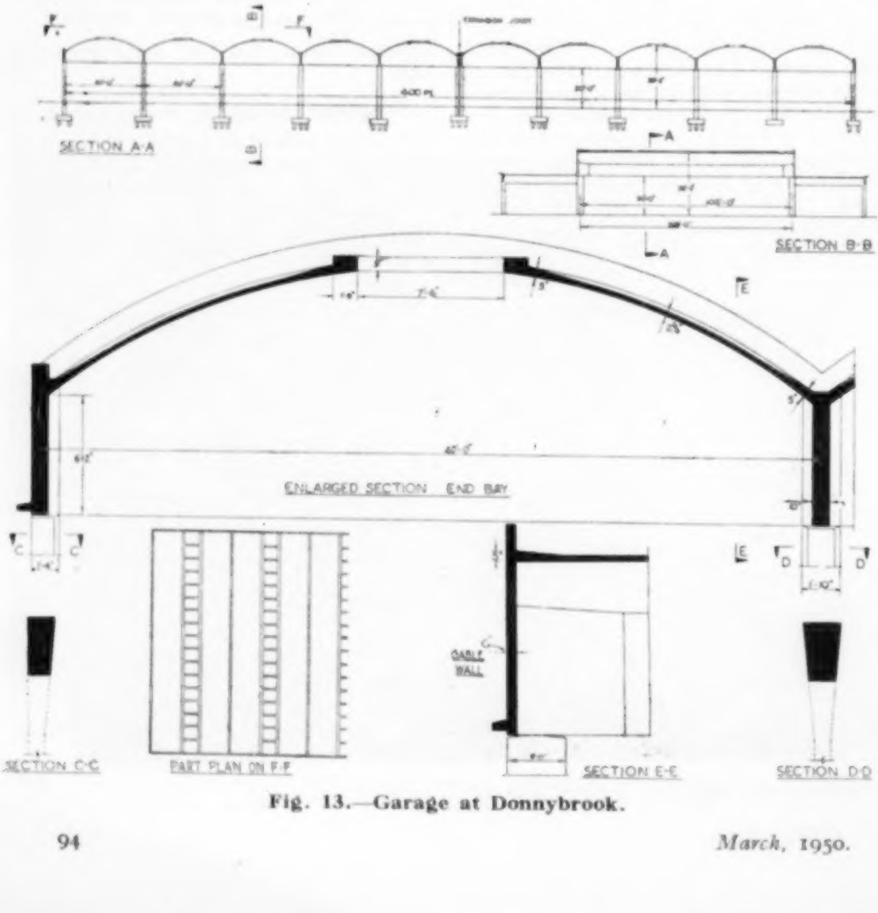

Fig. 13.—Garage at Donnybrook.

external walls will be tied to the columns by flexible ties thus giving the roof complete freedom to expand longitudinally.

The site is an old quarry filled in with various materials including in one part old tyres, which the pile tubes could not penetrate as they bounced off the rubber. The ground below the tyres and the layer of tyres, which occurs about 30 ft. below the surface, were consolidated down to the rock by injecting sand-cement grout, and cast-in-situ piles were formed in the upper 30 ft. Similar consolidation was also applied at another part of the site where loose granite setts had been deposited among other rubbish. The length of the piles varies from 16 ft. to 86 ft. The shorter piles are 15 in. diameter and the longer 19 in. diameter. A group of four or five piles is provided under each main column.

The columns are trapezoidal, thereby providing a space in which rainwater pipes are embedded. A hinge is provided between the top of the column and the longitudinal beam to prevent bending moment being transferred to the column as the beam deflected when the shuttering was removed. The resulting extension of the tensile reinforcement in the main beams, however, produces bending moments in the columns which are allowed for in the design of the columns, and the position of the columns on the pile caps is determined so as to produce nearly equal load on each pile. The main beams are 6 ft. 2 in. deep and generally 10 in. wide, but the width increases at each end to the width of the column so that the reinforcement is gently splayed out to pass the rainwater pipe. The reinforcement in the bottom of the beams comprises 24 1¼-in. bars, which includes some extra bars to form splices at the butt joints provided in the long beams. The stirrups are ½ in. diameter and are at 6-in. to 12-in. centres. The part of the rainwater pipe that passes through the hinged joint between the column and beam is a copper tube secured to the concrete on both sides of the hinge by brazed rings. The copper is ductile enough to take up any movement at the hinge. The gable walls are 6 in. thick and have a rib on the lower edge to weather the junction with the future enclosing walls.

The slabs of the vaults are generally 2¾ in. thick, splayed out to 5 in. for short distances at the springing, at the top-light trimmer beams, and at the junction with the gable walls. The struts across the light openings are 9 in. deep and 6 in. wide. The slabs of the vaults are rein-

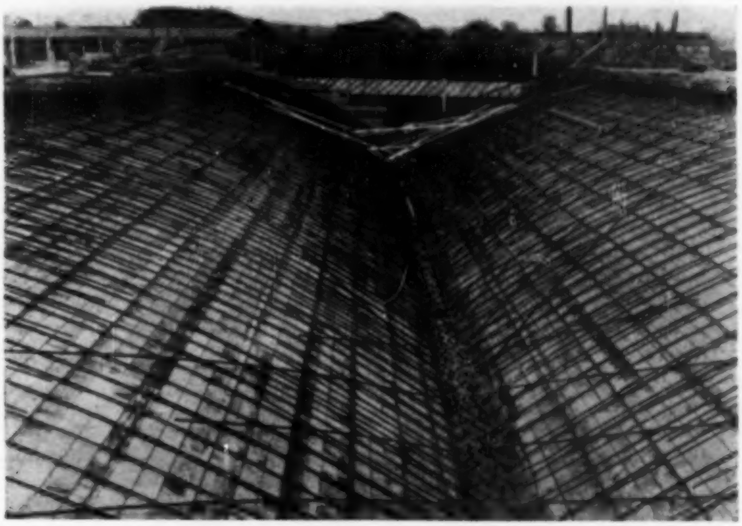

Fig. 14.—Garage at Donnybrook : Reinforcement for Roof.

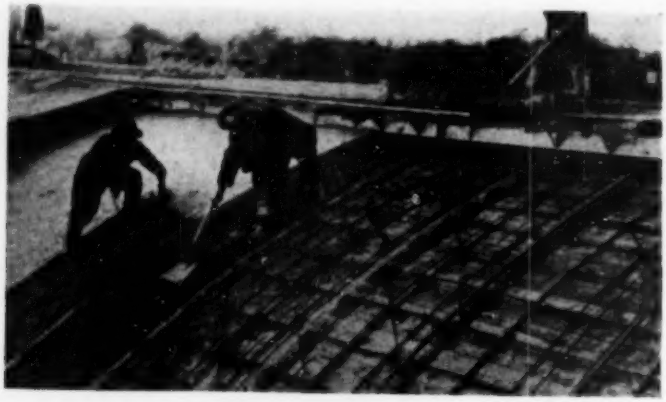

Fig. 15.—Garage at Donnybrook : Concreting the Roof.

forced with five layers of reinforcement (*Fig.* 15). Circumferential bars $\frac{3}{8}$ in. diameter are provided in the bottom throughout the length of the arc and for a short distance in the top at the springings and crown. Longitudinal bars $\frac{1}{4}$ in. diameter are provided between the $\frac{3}{8}$-in. bars at the bottom and top of the slab, and $\frac{1}{4}$-in. diagonal bars at varying pitch are provided in the middle of the slab. The thickness of the bars when rigidly assembled is $1\frac{1}{2}$ in. and therefore a cover of $\frac{1}{2}$ in. of concrete can be provided at the top and bottom in the $2\frac{1}{2}$-in. slab.

The beams were concreted in two operations, the first being the bottom part which contains a congestion of reinforcement. The mixture of the concrete in this part is 1 : $1\frac{1}{2}$: 3. Consolidation was by vibration. The second stage was to concrete the remainder of the beam, where the mixture is 1 : 2 : 4. The slabs of the vaults are of 1 : $1\frac{1}{2}$: 3 concrete containing $\frac{3}{4}$ in. ballast aggregate. A thin layer of 1 : $1\frac{1}{2}$ cement-sand grout was first deposited on the shuttering and this was followed by the stiffer concrete (*Fig.* 15). The slab of each half-vault was concreted in one operation starting from both ends at the same time. The concrete was laid in annular strips each about 3 ft. wide, concreting of each strip starting from the crown and proceeding towards the springing. Movable screeds, blocked off the shuttering, were used to give the correct thickness of slab.

The architects for the garage at Donnybrook are Mr. Michael Scott, F.R.I.A.I., and Mr. James Breman, M.R.I.A.I. The consulting engineers for the design and supervision of the structural work of the omnibus station at Store Street and the garage at Donnybrook are Messrs. Ove Arup & Partners. The contractors for the piling at Donnybrook garage were The Cementation Co. Ltd., and the contractors for the reinforced concrete work at the garage were Messrs. McNally & Co., Ltd., in collaboration with Messrs. Larsen & Nielsen A.S., of Copenhagen.

"Design of Reinforced Concrete Members in accordance with the British Standard Code."

It is intended to continue the publication of articles in this series in our next number.

Flat Roofs Covered with Water.

An economical method of constructing flat concrete roofs of industrial, agricultural, and residential buildings, which has been adopted in France, is to keep the concrete slab permanently covered with water, which is retained by low concrete parapets. By this method it is claimed that asphalt or other waterproof layer is unnecessary, since contact with the water stabilises the concrete and contraction cracks are thereby prevented. The water also acts as insulation, and other economies arise from the absence of slopes formed by screeding or otherwise, and the provision of only one down-pipe for cleaning and overflow purposes. The accompanying illustrations show roofs treated in this manner. *Fig.* 2 is an ordinary flat roof of beam-and-slab construction. The main beams of the roof of the building in *Fig.* 1 are bowstring girders projecting above the roof slab.

The depth of water required depends upon the climate, and in temperate climates would be about 1 ft. When low temperatures prevail for two or three weeks the water would freeze to a depth of, say, 9 in., but below the ice there would be water at 32 deg. F., and this would be the lowest temperature to

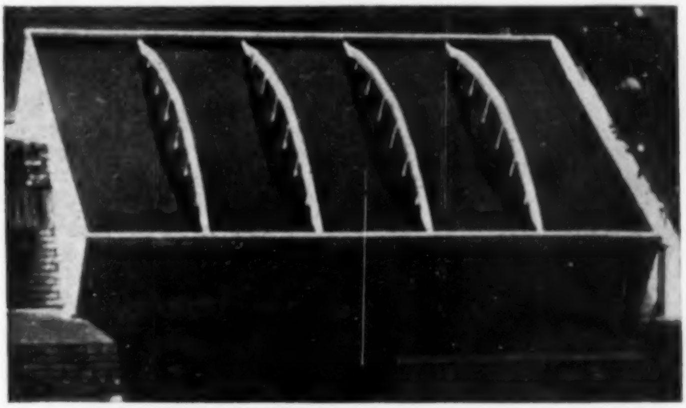

Fig. 1.—Roof Insulated with Water.

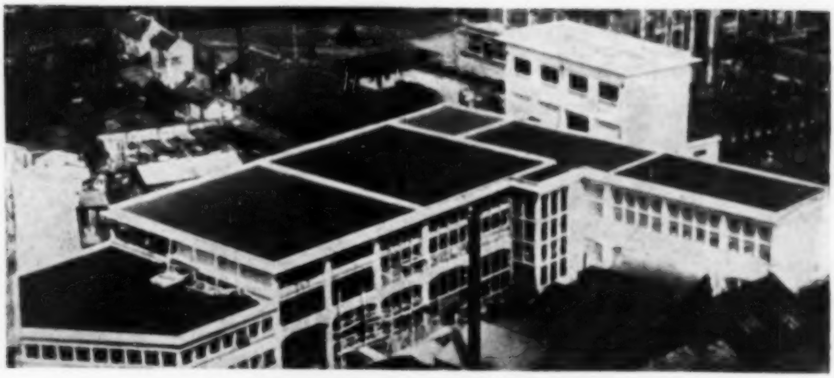

Fig. 2.—Roofs Insulated with Water.

which the roof would be subjected. The temperature of the surface of an ordinary roof exposed to air temperature which is for some length of time below freezing point, tends to approach this lower temperature. In hot weather the water keeps the roof cool, as the water becomes warm very slowly because evaporation tends to keep it cool. With an air temperature of, say, 120 deg. F. in the sun, the temperature of the water may reach 75 deg. F. Thus the seasonal variation of temperature is lower than is the case with an ordinary roof. There is practically no diurnal variation in temperature of rooms immediately below a slab covered with water.

The insulating value of the water also prevents extremes of expansion and contraction of the concrete and therefore there is less likelihood of cracks due to this cause and expansion and contraction in wet and dry weather respectively are also avoided. Snow and heavy rainfall are easily absorbed. There is, however, a small increase in the dead load which the slab and supporting members have to support, but as this is uniformly-distributed, that is of equal intensity on all spans simultaneously, the effects may be less severe than those of a smaller live load.

It is important that the concrete be well made and deposited with special care at construction joints, since leaks under a head of 12 in. of water may have disastrous results. Pure water should not be used because of its attack on the concrete. A little lime should be added to the water, when necessary, to ensure a *Ph*-value of about 6. It is recommended that the water should be run off once a year and the roof cleaned with a broom and clean water. In hot weather it may be necessary to maintain the depth of the water by adding water from a pipe and tap installed for the purpose. The fire-resistant properties of water-covered roofs are self-evident.

The foregoing notes were contributed by Monsieur G. Haymann, Ing. C. des Mines, who has been responsible for many buildings with roofs treated in this way. *Fig. 2* shows the building of the Compagnie Electron Industrielle at Fourichambault. *Fig 1* is a building at Nevers.

Prestressed Concrete Bridges in Yorkshire.

THE reconstruction in prestressed concrete of two road bridges over the Worsborough goods branch of British Railways, Eastern Region, was recently completed in connection with the Manchester-Sheffield-Wath electrification scheme. The sites of the bridges are at Gilroyd Lane and Hound Hill Lane, near Barnsley. The reconstruction was necessary because the old bridges provided insufficient clearance for the overhead conductors. The levels of the roads cannot be raised and the structural depth of the new work is therefore restricted. It is for this reason that prestressed concrete beams are used, as the thickness of the new deck, which spans 27 ft., is only 18 in.

The old masonry abutments have been rebuilt to give greater lateral clearances for rail traffic. As considerable subsidence of the ground is likely, the bridges are designed so that the decks can be jacked up when required to maintain their level. In the top of each masonry abutment three pockets are formed for the jacks, and the steel jacking beams, encased in concrete, form a bearing for the deck. The deck of each bridge comprises eighteen precast prestressed concrete beams 28 ft. 6 in. long. Each beam weighs about 25 cwt., and bears on steel plates on the jacking beams. The precast beams were erected by a 2-ton mobile crane (*Fig.* 1) which stood on the road leading to the bridge. The time taken to place each beam was about ten minutes. It was not necessary to close the line while the bridges were being erected, although it was necessary to do so while the old masonry arches were being demolished. Temporary bridges were erected to carry road traffic during the reconstruction.

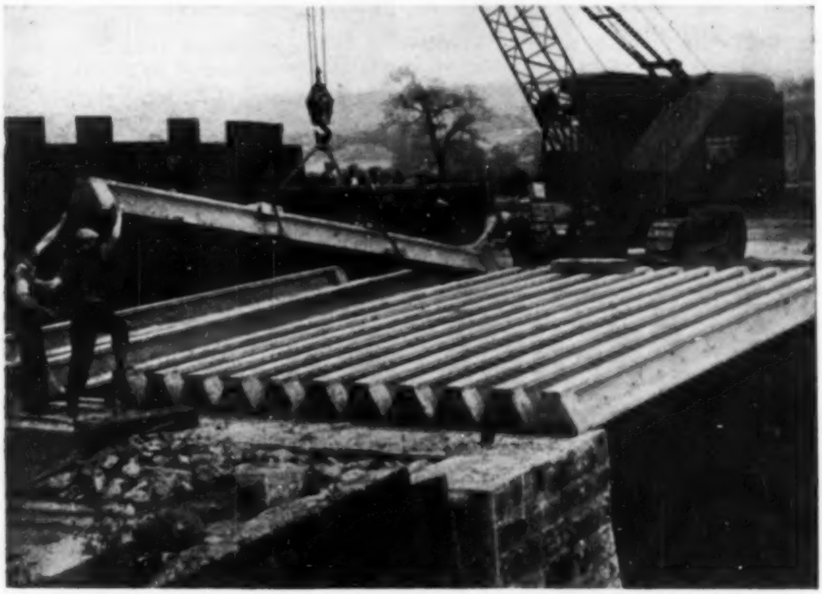

Fig. 1.—Erecting the Beams.

The beams are 18 in. wide at the bottom and 7 in. at the top, and are 14 in. deep. A description of the beams and of load tests made on a similar beam is given on page 52 of this journal for February, 1949. When the beams were in position, mild steel reinforcement was placed transversely across them and a cast-in-situ concrete slab 4 in. thick was

laid in a continuous operation to avoid construction joints. The composite structure thus formed, and the permissibility of small tensile stresses in the bottom flange of the beams under the greatest working load, enabled the overall thickness of the deck to be small. Precast concrete parapet slabs were erected later. The slab is covered with asphalt.

The bridges were designed and constructed under the supervision of Mr. J. I. Campbell, M.Inst.C.E., Civil Engineer, Railway Executive, Eastern Region. The prestressed beams were made by Dow-Mac Products, Ltd., at their works at Tallington, Lincs. The bridges were erected by Messrs. Wellerman Bros., Ltd.

The Effect of Fire on Prestressed Concrete.

Some tests were made recently at Maastricht, Belgium, to determine the resistance to fire of concrete beams prestressed with cables. A few previous tests on full-size beams made on the Continent gave satisfactory results within the range of temperatures generally expected when considering the resistance to fire of structures. The beam tested at Maastricht had a span of 38 ft. and was prestressed by a Magnel-Blaton cable. During the test the beam carried its working load of about 410 lb. per foot. The duration of the fire was 3½ hours and the greatest temperature recorded on the bottom of the lower flange was 1590 deg. F. The greatest additional deflection during the test was about ⅛ in., and the additional permanent deflection after the beam had cooled was negligible.

Beams and Columns Subjected to Twisting.

THE stresses in simply-supported beams and eccentrically-loaded columns subjected to torsion, and recommendations for the design of such members, are the subject of two recent publications of the Division of Building Statics and Structural Engineering of the Royal Institute of Technology, Stockholm. ("Combined Bending and Torsion of Simply-supported Beams of Bisymmetrical Cross Section," by O. Petterson. Price 3.50 kroner. "Torsional and Lateral Buckling of Eccentrically-compressed I- and Tee-Columns," by H. Nylander. Price 3 kroner. Obtainable from the Institute.)

In the former publication a non-linear theory of a common case of combined bending and torsion, namely, a simply supported bisymmetrical I-beam subjected to an oblique eccentric load at the middle of the span (a problem encountered in the design of crane girders), is derived.

The total stress in the longitudinal direction of the beam can be calculated from the stresses due to bending in two planes and to torsion by including terms which require the angle of rotation ϕ to be known. The differential equation for ϕ is solved for three cases, namely, an eccentric load parallel to the plane of maximum flexural rigidity, an oblique load passing through the centre of the cross section, and an oblique load applied at any point. The results are given in graphs from which the stresses can be calculated directly.

In the second publication it is recommended that eccentrically-compressed members of I- or tee-section in which the compressive force acts in the plane of the web should be designed so that the sum of the maximum stress due to compression and bending in the plane of the bending moment and a torsional stress is less than the allowable stress. The torsional stress is calculated on the assumption that the eccentricities at the top and at the bottom of the member are equal. An equation for use when the

eccentricities are not equal gives an adjustment factor. Equations for the torsional stress are given for the cases when the compressive load acts on the same or opposite side of the centre of twist as the centroid of the section, and involve the ratio of the yield-point stress to the allowable stress.

Although the analyses in the two publications relate to members of homogeneous material and the application is to steel beams, the theory could be extended to reinforced concrete members and especially to uncracked prestressed concrete members.

A Large American Concrete Mixer.

TILTING-DRUM concrete mixers of 6 cu. yd. capacity are being manufactured by the T. L. Smith Co., Milwaukee, U.S.A. These machines are said to be the largest concrete mixers yet made. They are operated by a 100-h.p. electric motor and, as they are not mobile, they are suitable for ready-mixed concrete plants and central-mixing stations on large construction works.

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Prevention of Corrosion of Reinforcement.

IT is stated by Mr. A. K. Datta in the Journal of the Association of Engineers (India) that rusting of reinforcement bars embedded in cement and lime mortars and in concrete is prevalent in the moist climate of Bengal. The corrosion occurs particularly in roof slabs, and also in reinforced brickwork. Prevention of this defect by increasing the cover of concrete over the bars or by increasing the cement content of the concrete would be costly. A method devised to prevent corrosion is to coat the steel bars with a paint composed of 1 part (by volume) of bituminous paint and 1½ parts of cement. The painted bars are then dusted with cement to provide a bond with the concrete.

Change of Address.

The office of Messrs. Ove Arup & Partners has been removed to 8 Fitzroy Street, London, W.1 (telephone, Langham 7781).

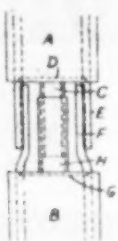
The offices and works of Cement & Steel, Ltd., are now at Second Avenue, Chatham.

Conversion Factors.

An addendum to British Standard No. 350 (1944), "Conversion Factors and Tables", has recently been issued and includes definitions of French units of force (sthene), pressure (pieze), and heat (thermie), and of the metric carat, the legal measure in Great Britain. Additional data of interest to civil and structural engineers include the conversion of cubic metres to acre-feet, and pounds per square foot to kilograms per square metre, and conversion factors relating to atmospheric pressure, thermal conductivity, conductance and flow, and traffic units. The table of linear measure (British to metric units) has been extended to 1000 ft., and examples are given showing how the use of the tables can be extended. An index to the data in the Standard and addendum is a useful addition. The addendum is issued by the British Standards Institution (Price 2*s*.).

Patent Relating to Reinforced Concrete Columns.

No. 591,431. Lee, D. H. May 7, 1945. Reinforced concrete columns (A, B) are interconnected by the engagement of axially-disposed metal tubes (C, H) projecting from their ends, and the column reinforcement bars (E, F) extend into the jointing space around the tubes. The



tubes may be a sliding fit or have a bayonet clip, set screws, or screw threads and each be secured to a metal plate (D, G) forming the end of the precast concrete members; the main reinforcing bars (E, F) may pass through holes in the plates and overlap or be bent to be concreted into an adjacent beam. The gap between the columns, which may accommodate reinforcement from one or more beams, is subsequently concreted. The tubes may be apertured to permit welding of the tubes or pressure grouting.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